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INVESTIGATION OF THE SEAFLOOR PRECONSOLIDATING FOUNDATION CONCEPT

By

P. J. Valent, D. A. Raecke, and H. G. Herrmann

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INVESTIGATION OF THE SEAFLOOR PRECONSOLIDATING FOUNDATION CONCEPT

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ABSTRACT

The report describes the performance of submerged model preconsolidating footings. The preconsolidation concept applied to seafloor footings can reduce the subsequent structure settlement by 80 to 95 percent, double the vertical load capacity, and it may double the lateral load capacity. Projected energy requirements of the pump system are compatible with the existing state-of-the-art. The concept will be useful for supporting sensitive and/or heavy structures on soft seafloor sediments.

INTRODUCTION

Seafloor installations may experience large total and differential settlements because of the presence of highly compressible cohesive sediments at many seafloor sites. Further, the potential is high for bearing capacity failures of spread footings on these soft sediments. Because of the likelihood that negatively buoyant seafloor installations will be emplaced in the future at deep-ocean sites with poor sediment properties, means for reducing or eliminating the potential settlements or bearing failures have been investigated.[1] One concept that was considered is the preconsolidation of the sediment beneath a foundation prior to emplacement of the supported structure. The technique of preconsolidation or precompression has been used often to stabilize construction sites for terrestrial structures.[2,3,4] The most common method of preconsolidation is the application of a static preload prior to construction. The preload must be left in place for a period of time to permit the settlement to occur. Usually the preload is of greater magnitude than the design structural loading in order to accelerate the rate of settlement and decrease the required consolidation time. An added benefit of preconsolidation is the accompanying increase in soil shearing strength which improves the bearing capacity and lateral load resistance of the foundation.

Because of the difficulty of using a deadweight preloading method in the deep ocean, the concept that was developed consists of inducing a negative relative pressure in the pore water in the sediment. This increases the particle-to-particle effective stress and causes consolidation.[1,4] Figure 1 shows the essential features of the concept. The footing encloses a porous filter material within an impervious keying edge or skirt. The pump applies a suction to the filter and induces the negative relative pressure in the sediment. An accumulator between the pump and the interior of the footing maintains the negative pressure and allows the pump to be run intermittently. The flow of water toward the filter will begin to establish a pattern similar to the flow net in Figure 1. In the period of transition from no flow to fully established flow (as represented by the flow net), the effective stresses in the sediment are increased and consolidation occurs. In effect, a portion of the weight of the water column above the footing is added as a surcharge load to the footing. After the desired degree of preconsolidation is achieved, application of the negative relative pressure to the footing underside may be discontinued, since the soil consolidation process is largely irreversible (about 90% of the induced volume change is permanent).

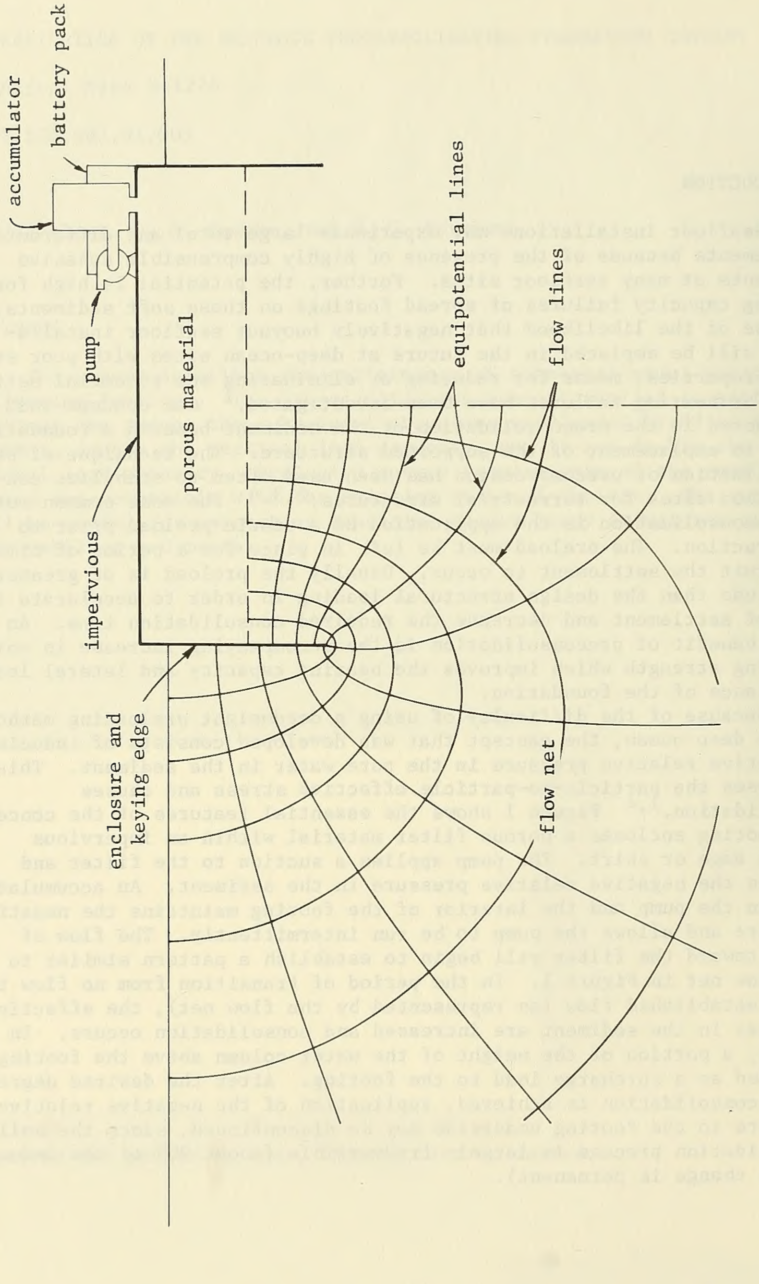

Figure 1 Schematic of preconsolidating footing

This report describes a laboratory evaluation of the benefits that may be derived from the preconsolidation of seafloor footings. A supporting analytical evaluation is described in Appendix B.

LABORATORY INVESTIGATION

The major purpose of the model testing program was to verify the improvement in sediment behavior due to preconsolidation by the induced negative pore pressure technique. Of particular interest was the degree of reduction of long-term settlement under design load. The increases in bearing capacity and resistance to lateral loads were also of major interest. A secondary purpose of the model testing was to provide data on the influence of the skirt-depth/footing-diameter ratio on preconsolidation settlement and lateral load capacity. The latter information will be useful for the economical design of prototype footings.

Test Program

The model tests were conducted in two bins, each 4-feet on a side by 2½-feet deep, containing artificially prepared soil simulating seafloor sediments. Two soils were used: a low-plasticity silt from Seal Beach, California; and a low-plasticity clay from Rogers Lake, California. (See Appendix A for detailed properties.) The soils were placed in the bins as slurries and were consolidated by drainage to a thickness of approximately 13½ inches. The majority of tests were conducted on the Seal Beach silt. Tests were conducted on the Rogers Lake clay to confirm that the preconsolidation concept worked similarly in a very fine-grained sediment.

A typical model preconsolidating footing (see Figure 2) consisted of a filter stone with its top face and sides enclosed by 1/8-inch thick sheet steel. This sheet extended some distance below the filter stone to lengthen the pore water drainage path. A vacuum hose connection on the footing top penetrated the sheet steel to drain the filter stone.

Three sizes of model footings were used to investigate the scale effect: two, six, and 12 inches in diameter. Two ratios of keying edge depth to diameter, 1:8 and 1:4, were employed to evaluate the effects on settlement and lateral load resistance. The preconsolidation pressure applied to the footings was usually 10 psi, with 5 psi used in a few tests for comparative purposes. These negative pressures were applied until measurable vertical movements ceased (assumed completion of primary consolidation). Similarly, most non-preconsolidated footings were not tested until they had ceased measurable movement under the applied loadings, although a few tests at partial consolidation were conducted to determine initial bearing capacity and lateral load resistance.

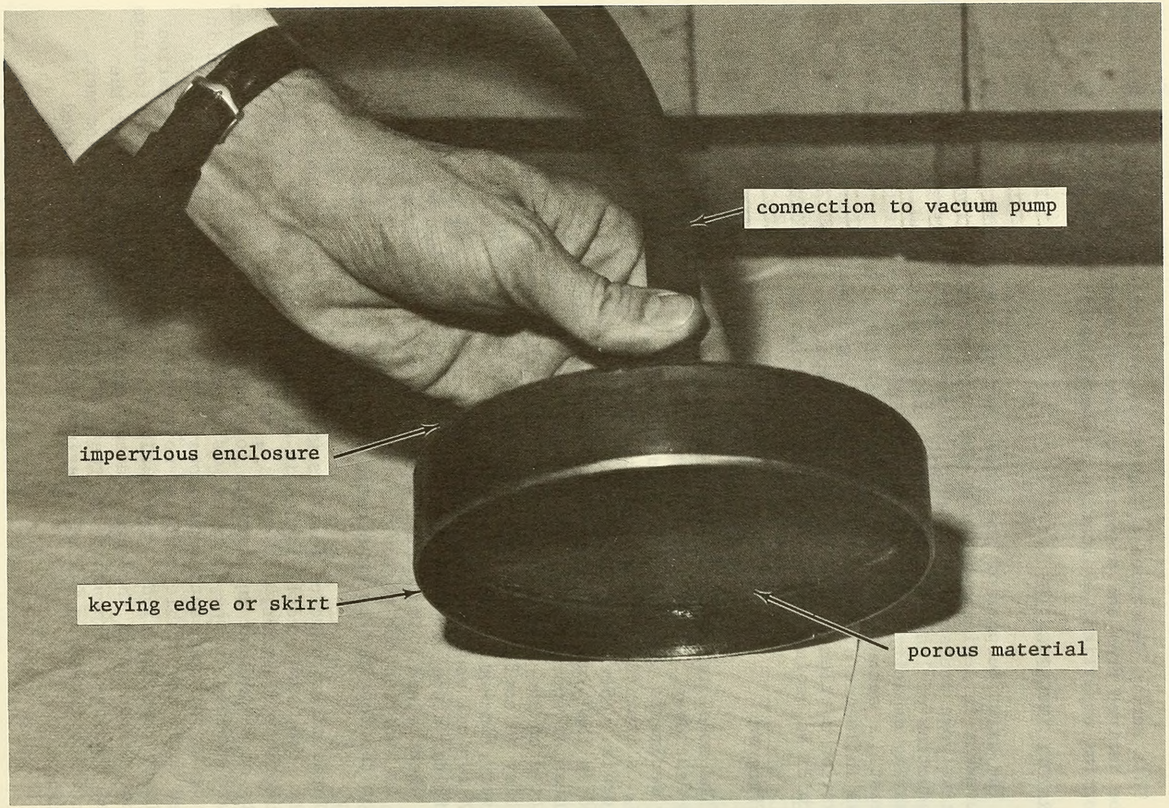

Figure 2. Model preconsolidating footing.

Test Procedures

Following the soil preparation, as described in Appendix A, the model footings were placed on the soil surface in specified patterns designed to minimize mutual interference. The descent rate of the footings was controlled to give the water trapped between the footing and soil surface sufficient time to escape and to minimize disturbance of the soil. Initial measurements for settlement records were taken as soon as the soil was supporting the weight of the submerged footing.

The subsequent treatment of each footing was a function of the tests scheduled for it. Some non-preconsolidated footings were tested for immediate bearing capacity and lateral load resistance, usually within 60 minutes of the time the footing was placed on the soil. The remaining non-preconsolidated footings were allowed to reach complete primary consolidation settlement, indicated by cessation of measurable displacement, under the dead load of the footing prior to further testing. The footings to be preconsolidated were connected to a common vacuum manifold and vacuum was applied within 24 hours after the footings were emplaced. These footings were also allowed to reach complete primary consolidation settlement prior to testing. Figure 3 shows a typical laboratory test arrangement. Three 2-inch and two 6-inch diameter footings are connected to the vacuum manifold ready for preconsolidation, and one 2-inch and one 6-inch diameter footing are settling under their own dead load. The settlement of the footings was measured with the simple rod-and-scale device shown in Figure 3, and with a dial gage system not shown. With the rod-and-scale device, settlement readings could be estimated to the nearest 1/64 inch. The dial gage system was an order of magnitude more accurate than the rod-and-scale system but was not as versatile since the dial-gage spring force frequently caused extra movement of the small footings, particularly in the Rogers Lake clay.

When several of the non-preconsolidated and preconsolidated model footings had ceased settling, they were subjected to an added "design loading" which amount to ¼ pound for the 2-inch footings, 3 1/3 pounds for the 6-inch footings, and 13 1/3 pounds for the 12-inch footing. These design loadings applied a pressure increase of about 0.12 psi to the soil surface. (The footings themselves applied about 0.06 psi.) The total design settlement loading; i.e., footing dead load plus design load, had a factor of safety of about two with respect to the experimentally determined bearing capacity of the non-preconsolidated footings.

MODEL PERFORMANCE

Settlement

The effect of preconsolidation alone on the model footings is shown in Figures 4 and 5. Figure 4 illustrates typical performance of

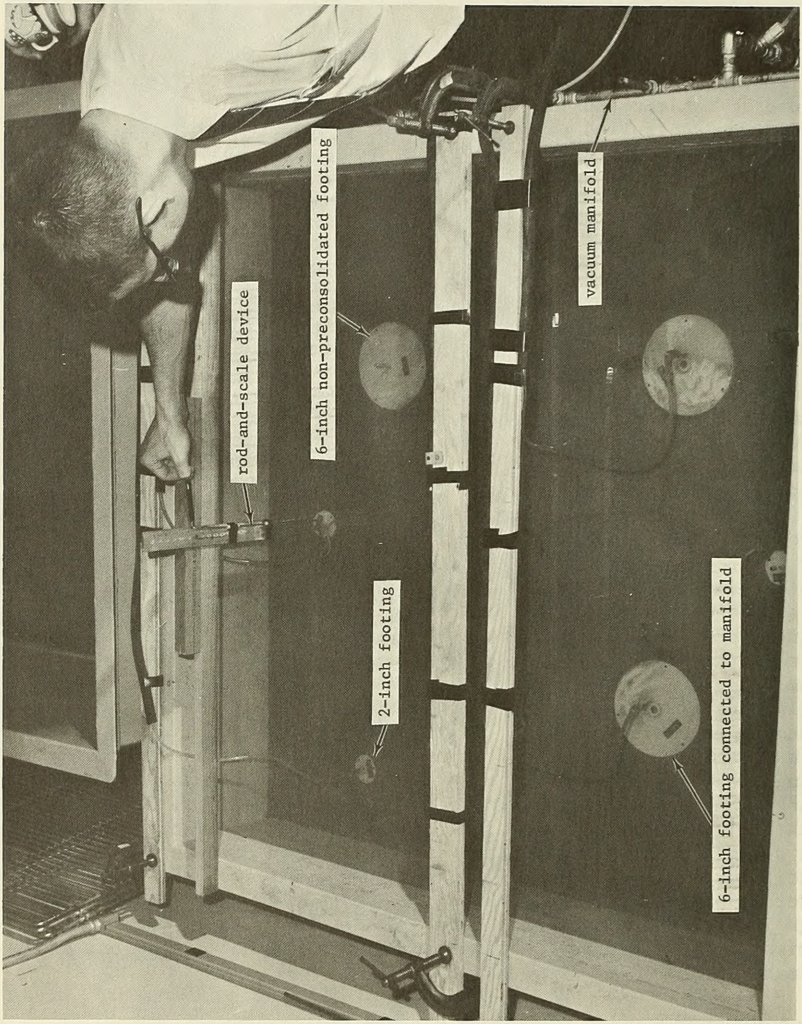

Figure 3. Typical laboratory test arrangement.

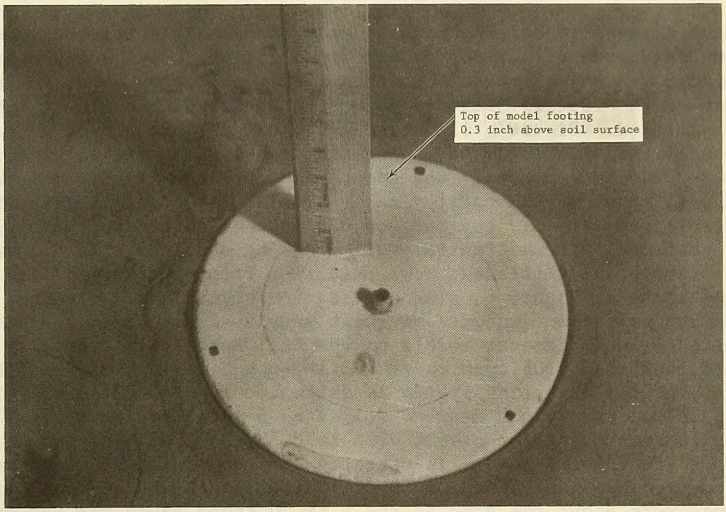

Figure 4. Six-inch non-preconsolidated footing after settlement due to footing load.

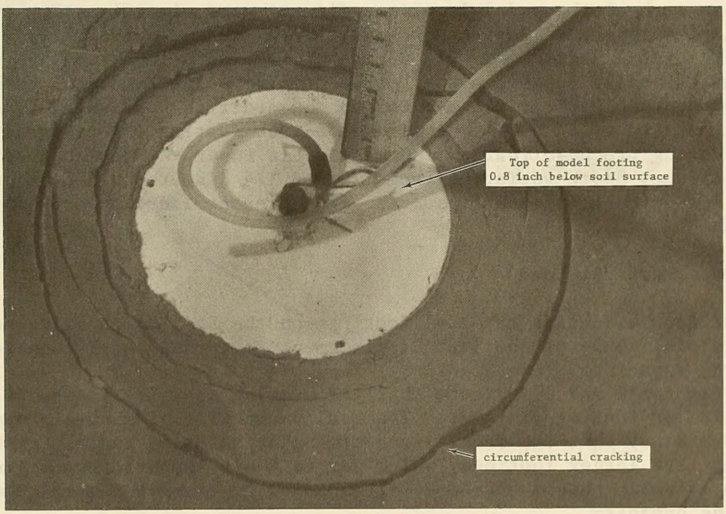

Figure 5. Six-inch footing after preconsolidation (vacuum 10 psi).

a 6-inch diameter model which has ceased measurable settlement due to its own dead load of 1.8-pounds submerged weight. Such settlement averaged 0.3 inch. Since the thickness of the footing was 0.63 inch, the top of the footing remained above the surrounding sediment surface as illustrated. Figure 5 illustrates typical performance of a 6-inch preconsolidated footing which has ceased measurable settlement due to its 1.8-pound buoyant dead load and to the applied negative pressure of 10 psi. Such settlement typically totaled 1.4 inch; thus the footing stabilized with its top surface 0.8 inch below the surrounding sediment surface. Circumferential soil cracking outlines soil "slump blocks" above the settlement bowl resulting from preconsolidation.

As shown in Figure 6, preconsolidation significantly reduces foundation settlement under subsequent gravity design load. Preconsolidated 6-inch diameter models on the Seal Beach silt typically settled less than 0.05 inch under a 3 1/3-pound design loading (the accuracy of the measuring system is estimated to be 0.02 inch). The same load applied to a non-preconsolidated 6-inch footing typically led to long-term settlements of 0.25 to 0.65 inch, or settlements 5 to 20 times greater. Figure 7 illustrates that the ratio of settlements for non-preconsolidated and preconsolidated 2-inch and 12-inch diameter footings were about the same as the 5 to 20 times found for the 6-inch models. The wide variation in the settlement performance of non-preconsolidated models, for example, curves one and two of Figure 6, is not adequately understood. The initial condition of the supporting soil at each of the model locations did not vary significantly, and model placement and loading procedures were carried out similarly.

The total settlement of non-preconsolidated 6-inch footings included typically 0.30 inch due to the footing weight and 0.65 inch due to the design loading, for a total on the order of 0.95 inch. The total settlement of a preconsolidated 6-inch footing was typically 1.40 inch during preconsolidation and negligible under the design loading.

The key depth variation from 1/8 and 1/4 diameter had no significant effect on settlement resulting from preconsolidation or design loading.

Ultimate Vertical Load

Preconsolidation of model footings on the Seal Beach silt reduces the initial slope of the load-displacement curve and nearly doubles the bearing capacity. (See Figure 8)

Figures 8 and 9 indicate that the bearing capacity increases as a function of footing diameter. The Seal Beach silt bins were prepared in such a way that the density and strength increased rapidly with depth. The larger footings, which derive their support from a greater depth of soil, should therefore be expected to exhibit a greater unit bearing capacity.

In general, footings failing in bearing capacity remained nearly horizontal, and the displaced soil exhibited itself as a uniform

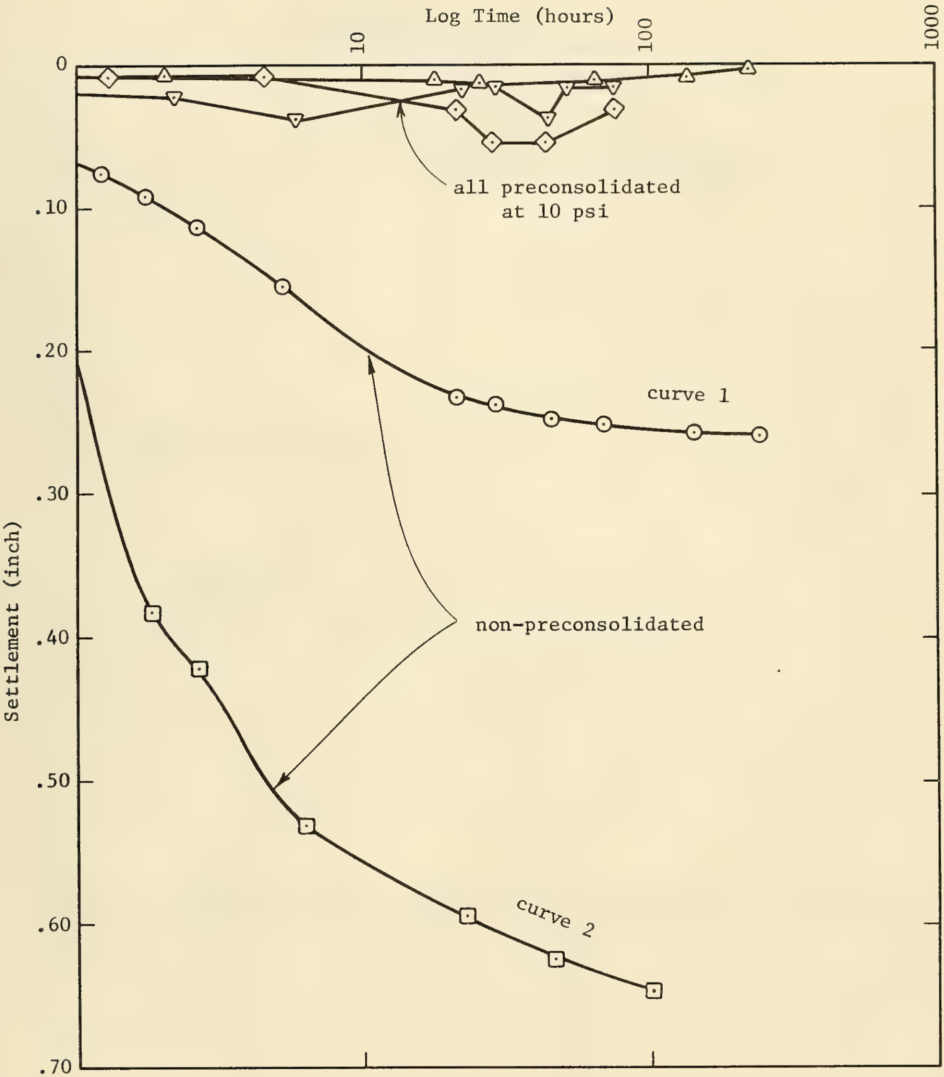

Figure 6. Settlements of typical non-preconsolidated and preconsolidated 6-inch diameter footings due to 3 1/3-pound design loading, on Seal Beach silt.

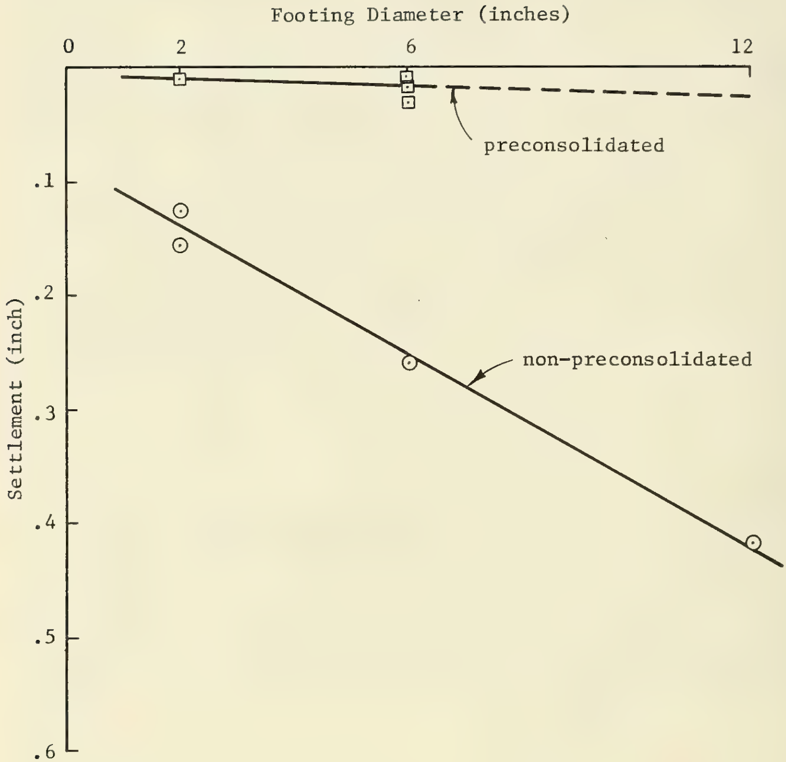

Figure 7. Influence of footing diameter upon settlement due to design loading (about 0.12 psi), on Seal Beach silt.

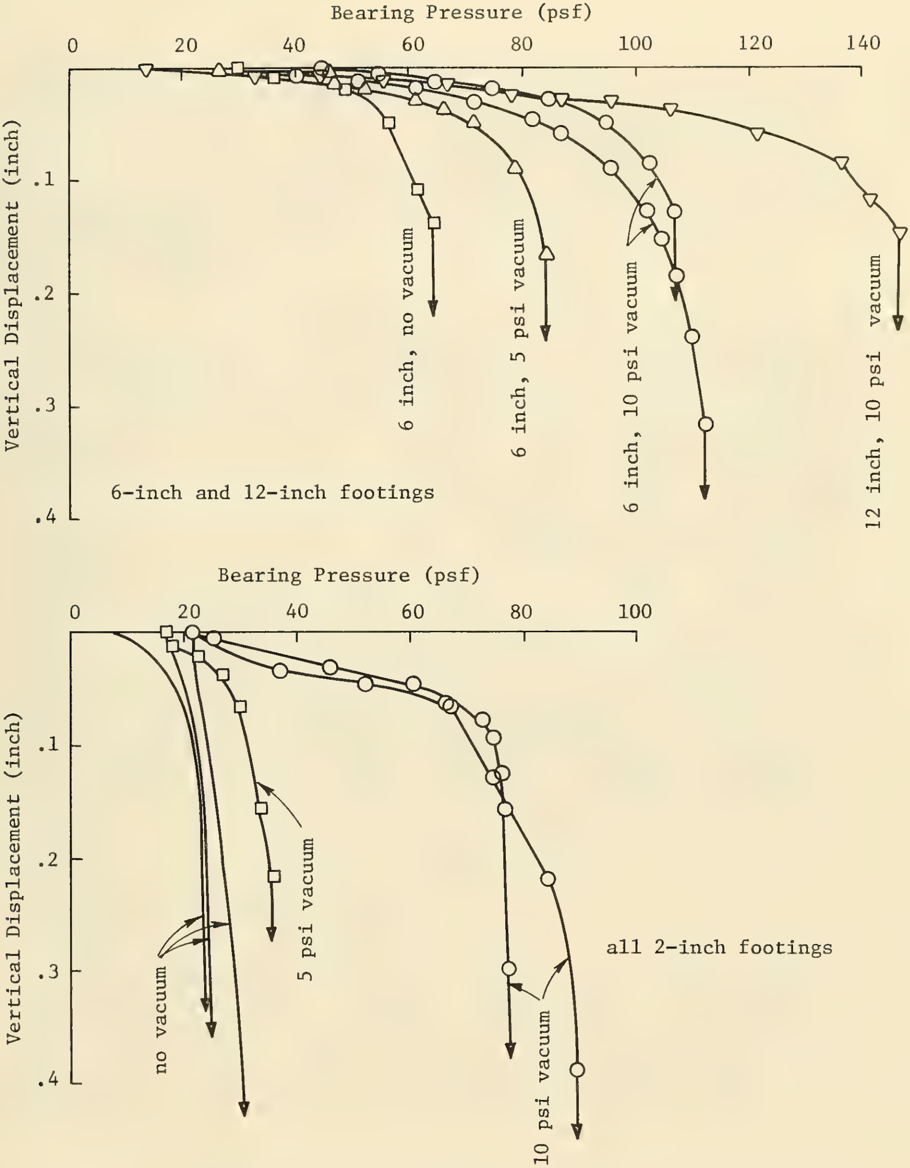

Figure 8. Vertical load-displacement curves for non-preconsolidated and preconsolidated footings, Seal Beach silt. For all curves, skirt depth is 1/8 footing diameter.

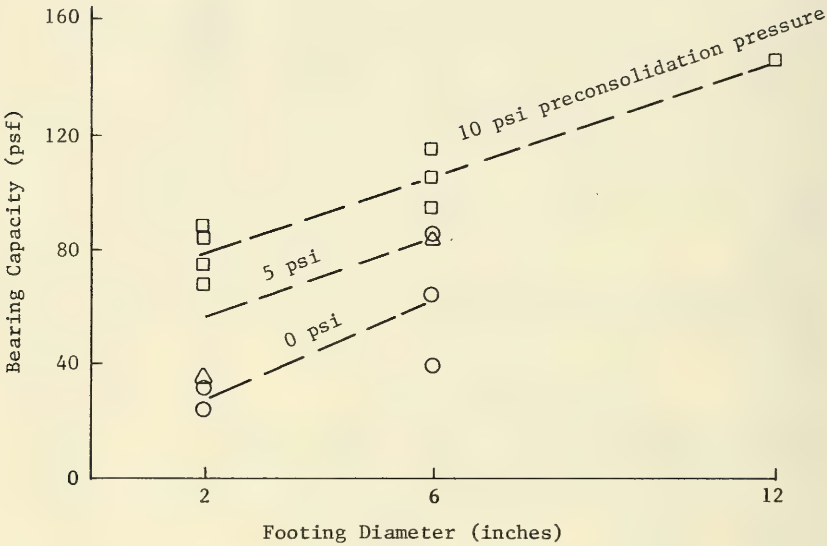

Figure 9. Effect of footing diameter upon bearing capacity, Seal Beach silt.

heaving about the footing perimeter. Occasionally a footing would tilt preferentially during the latter loading increments, but still no evidence would appear of a distinct failure plane intersecting the soil surface. Footings which had been preconsolidated and then failed in bearing were found to have a "cone" of consolidated soil adhered firmly to their bottom. (See Figure A-2 for condition of soil in these cones.)

Key depth variation had no significant effect on performance during vertical loading to failure.

Lateral Load

Lateral load was applied to model footings at the center point of the top surface, whereas the resultant force vector resisting this lateral load operated at some elevation below the footing top surface. Thus, the laterally loaded footings were subjected to an overturning force couple during lateral load testing. The footings were free to rotate and/or rise up out of the soil; in most tests the design vertical load was maintained during lateral loading to inhibit this tendency to rise up. The exact magnitude of improvement in lateral load capacity, resulting from preconsolidation, is not clear from the model test data; however, it can be said that the improvement is probably not greater than two-fold. (See Figure 10.)

Non-preconsolidated footings failed in the plane of the footing cutting edge. Preconsolidated footings on the Seal Beach silt exhibited a complex failure mode: a non-symmetric wedge of soil adhered to the model base with a shallow, long shear plane on the leading edge, and a steep, short plane of tensile separation on the trailing portion of the wedge. The lateral load failure mode of footings in the Rogers Lake clay was considerably different. The preconsolidated footings failed by rotation about a point about a half-diameter beneath the footing; excavation of one model revealed a spheroid of soil adhered to the model base. The model and soil spheroid rotated within the soil mass like the ball of a ball-and-socket joint. The change in failure mode between the Seal Beach silt and the Rogers Lake clay is attributed to the difference in the soil strength-depth profile: strength in the silt increased rapidly with depth while strength in the clay was much lower and increased less rapidly with depth.

Skirt depth variation did have a net effect on lateral load capacity with the deeper skirts offering much better performance. (See Figure 10, 10 psi curves.) Increased capacity due to increased skirt-depth/footing-diameter ratio derives primarily from the larger passive wedge being pushed in front of the footing and/or from possible increased soil strength on the failure planes beneath the footing. The mechanism responsible for the improved performance has not been identified.

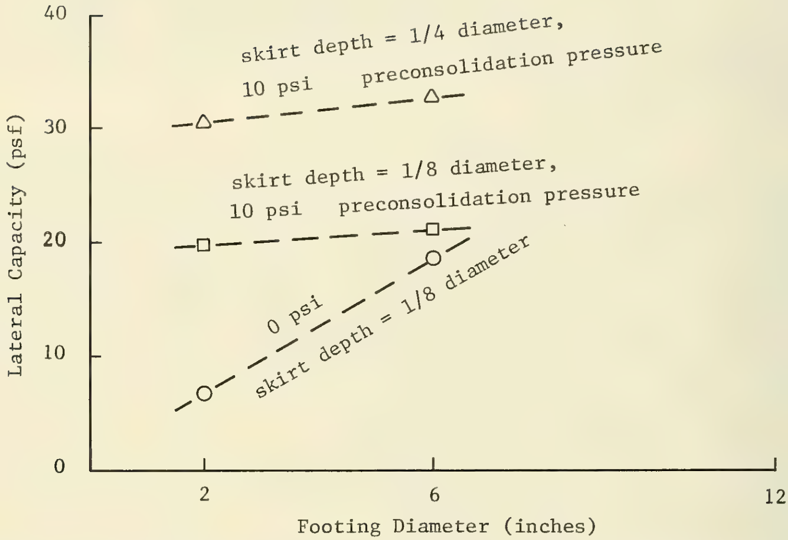

Figure 10. Effect of skirt depth/diameter ratio, preconsolidation pressure, and footing diameter upon lateral capacity, Seal Beach silt.

Skirtless Preconsolidating Footing

Emplacement of a preconsolidating footing with vertical skirts must include expulsion of water trapped by the skirts between the footing base and the sediment surface. Expulsion has been accommodated in similar full-scale situations by fitting the foundation with vent pipes with flap valves such that water can pass out but not back in. Experience with the models has shown that these vents must have a high flow capacity because otherwise the trapped water will exit around the skirts eroding channels and also may cause the soil beneath the footing to lose strength and flow. This often resulted in excessive footing tilt.

To eliminate the water exit problem, two attempts were made to eliminate the vertical skirt and replace it with a horizontal skirt extending out to 9-inches diameter (on a 6-inch diameter model). This modification was unsuccessful on a model scale because of the difficulty of sealing the horizontal skirts (membranes) to the soil surface.

Applicability to Full-Scale Foundations

This laboratory model study and the supporting analytical study (see Appendix B) have indicated that the preconsolidation concept applied to seafloor footings can reduce structure settlements to 1/5 to 1/20 that of conventional, non-preconsolidated footings; and preconsolidation can, at the same time, double the ultimate vertical and horizontal load carrying capacities.

Such performance improvements are valuable for installations sensitive to large total settlements and/or large tilt, for instance, equipment emitting a directional beam or equipment intended to locate a sound source. Often seafloor foundation size is dictated by the space available on the launch vessel and/or by the emplacement-line load capacity, and the engineer is faced with the problem of making a too-small foundation adequate. Here, preconsolidation may be of benefit by increasing vertical and/or lateral load carrying capacity.

The preconsolidating footing also offers another benefit: the resistance of the footing to long-term pull-out loads is considerably improved. Thus, the preconsolidated footing, as long as the "suction" is maintained, will provide improved overturning resistance and may even be used as a tie-down.

The seafloor preconsolidating footing concept does have a drawback: the footing and supported structure must normally be placed separately. The footing must be installed first and the preconsolidation pressure applied for a time period on the order of one month prior to application of the structure load. Thus, two separate sea operations are typically required. In addition, the structure emplacement operation requires the capability to mate the structure to the footing on the seafloor. But despite these drawbacks, the preconsolidating footing concept remains viable because alternate foundation systems capable of supporting heavy seafloor installations, with little total settlement and tilt,

require still more complex state-of-the-art advancements, for example, an underwater pile emplacement system.

The necessary pump, accumulator, switching apparatus, and battery package required for a 10-foot diameter prototype are within the existing state-of-the-art. Measurements were made of the quantity of water pumped through some of the models under a head of 10 psi. The power involved in expelling this water is about 0.002 watts for the 12-inch diameter footing. The requirements of a 10-foot diameter prototype footing with a pump system operating at an efficiency of 10 percent could be supplied by 14 pounds of lead-acid automobile storage battery. This evaluation presumes the availability of specially designed equipment. An alternate, empirical approach to determining the power requirement, using off-the-shelf equipment, indicated a need for 120 pounds of lead-acid storage batteries. Thus, the concept is workable.

CONCLUSIONS

1. The seafloor preconsolidating foundation concept can be effectively used for the support of sensitive and/or heavy undersea structures on seafloor sediments.

2. Laboratory model tests and supporting analytical work indicate that use of the concept reduces the structure settlement by 80 to 95 percent and doubles the vertical load capacity. Comparable improvements in performance are expected for prototype in-situ foundations.

3. Laboratory model tests indicate that use of the concept does increase the foundation lateral load capacity, possibly doubling the magnitude. The present failure model used for foundation lateral load capacity prediction is not applicable to preconsolidated foundations. Research is needed to develop such a predictive capability.

Appendix A
SOIL BIN PREPARATION

The Seal Beach silt was obtained from a coastal mud flat in the Long Beach, California, area. The material had a liquid limit of 49; a plasticity index of 20; Unified Classification, ML; and Trilineal Classification, clayey silt. The Rogers Lake clay is a commercial product generally used as a drilling mud. The clay had a liquid limit of 48; a plasticity index of 21; Unified Classification, CL; and Trilineal Classification, clay. Grain size curves are presented in Figure A-1.

Two test bins were used, each 4-feet on a side by 2½-feet deep. A 3-inch thick porous sand-cement blanket was compacted and cured in the bin bottom and connected to drain valves on the outside bottom of the bins. The permeability of this filter was several times greater than that of the test soils.

The Seal Beach silt was prepared initially in a vertical two-beater mixer with close-fitting container. The clayey silt was added to the container followed by chlorinated seawater until a pourable slurry, free of lumps, had been attained; then mixing was continued for five minutes while a 25-inch Hg vacuum was applied to remove air bubbles. The de-aired slurry was poured into the test bin through one inch of seawater. Approximately 20 inches of de-aired slurry were placed in the first bin. The silt slurry was consolidated by applying a 9-foot head of water to the material via a siphon. Soil surface settlement reached a negligible rate within three weeks, after which the bottom drainage was discontinued, and the soil-water system was allowed to reach equilibrium for two days. Final soil depth was about 13½ inches.

At the completion of the first and second test series, the Seal Beach silt was covered with ten inches of seawater, and then was stirred with a half-inch electric drill and paint mixer type blade to return the soil to its original pourable slurry state. The consolidation process using the siphon was then repeated. The mixing process did entrain some air bubbles, such bubbles could be seen breaking at the slurry surface.

The Rogers Lake clay was mixed in a double planetary mixer without vacuum de-airing to a pourable consistency. The as-delivered clay product consisted primarily of dried clay lumps; these lumps were broken down by mixing in tap water with calcium hexametaphosphate added as a dispersant. Subsequently C.P. grade sodium chloride was added to the slurry in the amount of 35 ppt. The method of placement of the clay slurry in the second bin was the same as for the first placement of the clayey silt except that a number of air bubbles could be seen breaking at the surface. No seepage force was applied to the clay; instead the pressure head at the base of the soil was maintained at the level of the water surface over the soil. The purpose of this procedural change was to obtain a soil strength versus depth profile which

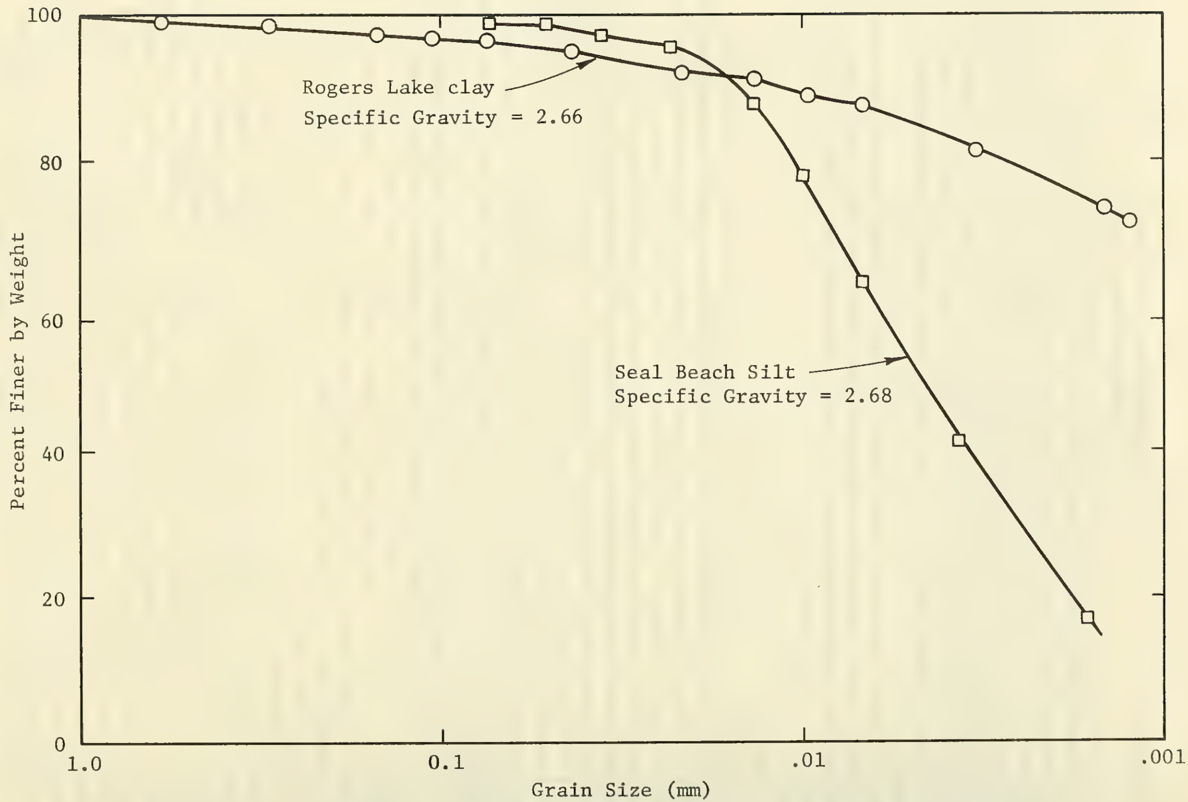

Figure A-1. Grain size curves of test soils.

increased less rapidly with depth compared to the Seal Beach silt strength profile.

Measurements of soil water content and vane shear strength were made before testing and after testing beneath the footing centerlines. (See Figures A-2 and A-3.)

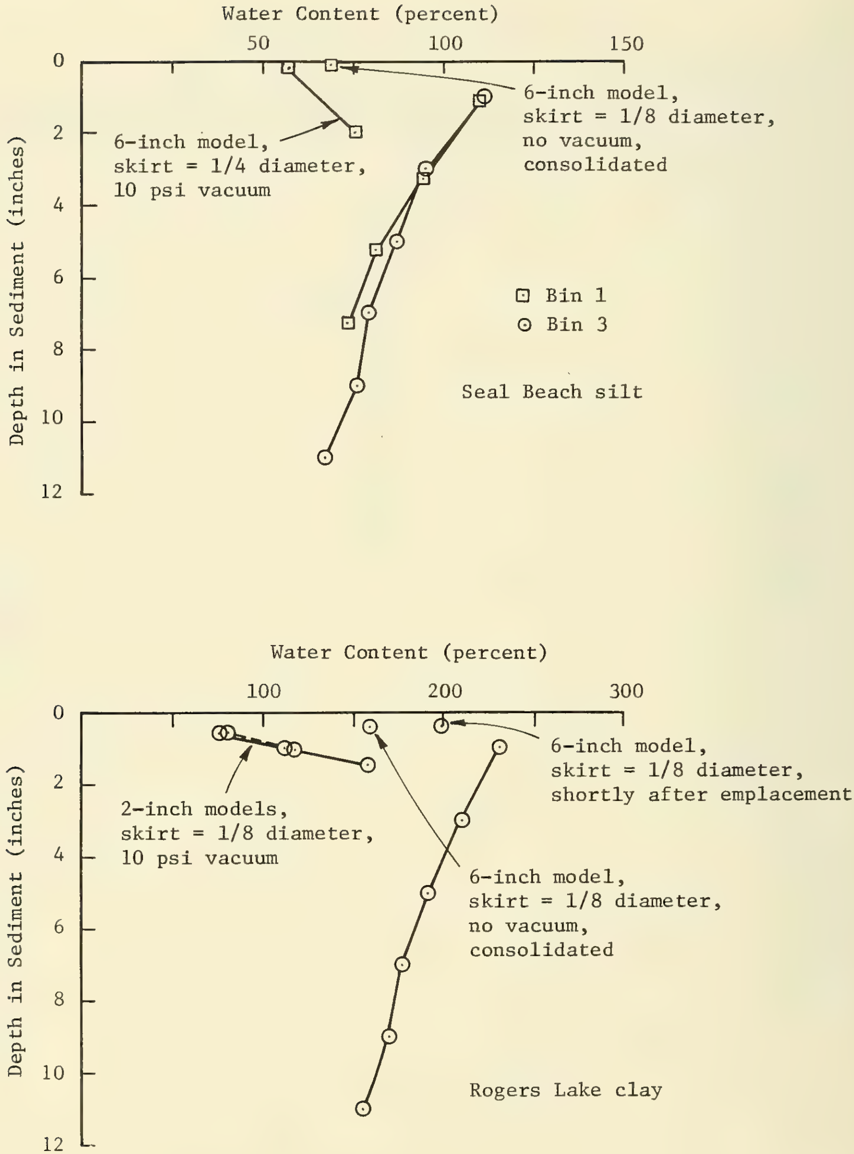

Figure A-2. Soil water contents in test bins including some water contents from immediately beneath the footings.

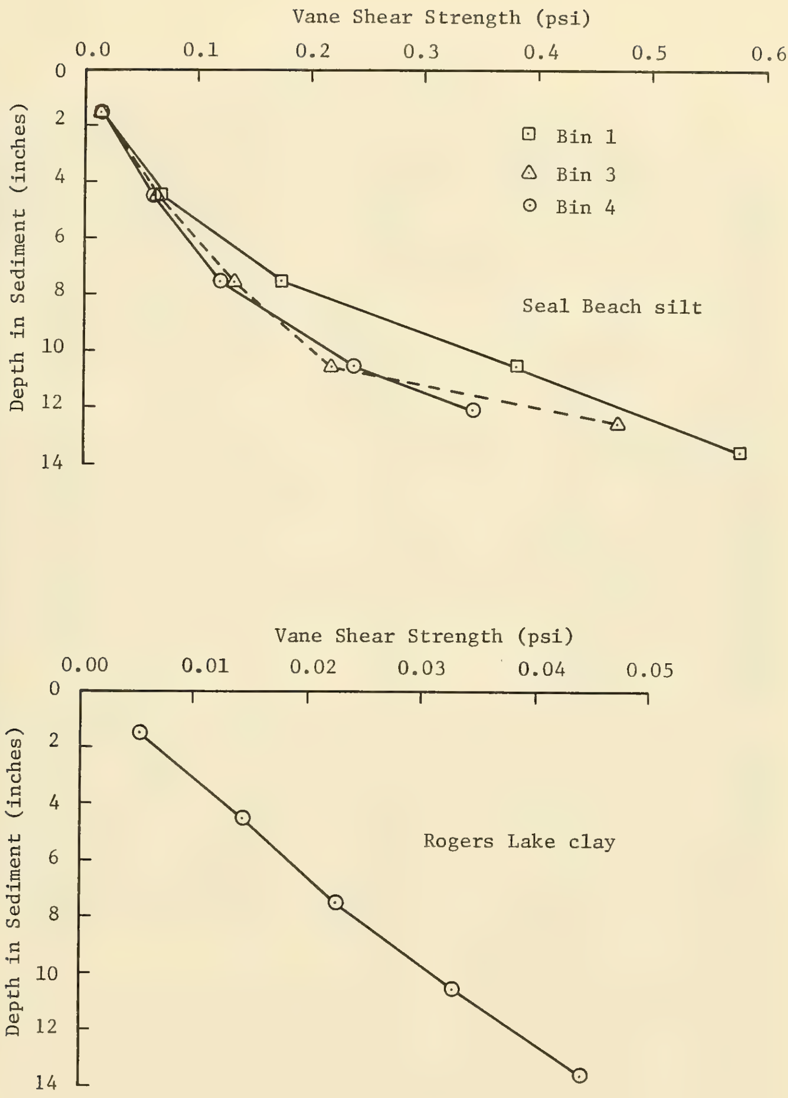

Figure A-3. Soil vane shear strength in test bins.

Appendix B
ANALYTICAL INVESTIGATION

An analysis using soils data from two thoroughly investigated NCEL test sites was made to determine the benefits of using full-scale preconsolidating footings. Both sites are located in the Santa Barbara Channel. The first is in 120 feet of water off Pitas Point; the soil classified as ML by the Unified System and as clayey-silt by the Trilineal System. It has a liquid limit, LL, of about 40; plasticity index, PI, about 15; compression index, $C_c = 0.30$; and coefficient of consolidation, $c_v = 0.007 - 0.012\ cm^2/sec$. The second site is in 1200 feet of water near the center of the Channel; the soil here is classified as MH by the Unified System and clayey-silt to silty-clay by the Trilineal System. It has a LL of about 92, PI about 49, C_c about 0.77, and $c_v = 0.00075 - 0.0012\ cm^2/sec$.

The additional effective stresses in the soil resulting from an applied negative pore water pressure were evaluated from a two-dimensional flow net (see Figure 1). This flow net assumed a homogeneous isotropic section. The vertical effective stress component from the two-dimensional analysis was then applied directly to the three-dimensional problem; the resulting errors were neglected. Void ratio change estimates are therefore based on one-dimensional consolidation. Shear strengths for the new void ratios were projected from the in-situ e-log p relationship. Predictions were made of the effects of a 10-psi preconsolidation pressure applied to a 10-foot diameter weightless footing (see Table B-1).

The settlement reductions and bearing capacity increases predicted for the real soil profiles were of the same relative magnitude as those from the laboratory model tests. For instance, at Pitas Point, the analytically determined settlement of a preconsolidated footing due to design load is shown to be zero compared to 4.5 inches for the non-preconsolidated footing design load settlement (see Table B-1). Allowing for the inaccuracies of the laboratory measuring system, this settlement reduction compares fairly well with the minimum 80 percent reduction in laboratory model settlement. Preconsolidation increased the analytically determined bearing capacity 2.8 times, from 380 psf to 1060 psf, which compares favorably with the minimum two-fold increase found in the laboratory.

Lateral load capacity increases predicted for the real soil profiles differed in relative magnitude from the laboratory model test results. For the Pitas Point profile, analytical lateral load predictions indicated a 6.6 times increase, from 5,600 pounds to 37,000 pounds; whereas model results in the bins showed only a two-fold increase. The discrepancy probably largely arises because (1) the assumed analytic failure plane was in the horizontal plane of the cutting edge, which was not the case in the laboratory tests, and (2) the test footings were subjected to an eccentric lateral load (above the cutting edge plane) which resulted in tension stresses at the footing heel.

Table B-1. Theoretical Performance of 10-Foot Diameter Preconsolidating Footing at Two Sites

Performance Component	Time Duration or Degree of Preconsolidation			
	None	15 days at 10 psi	60 days at 10 psi	Complete* at 10 psi
Pitas Point				
Settlement due to 210 psf design load (in)	4.5	0.25	0	0
Bearing capacity (psf)	380	740	920	1,060
Lateral resistance (lbs)	5,600	29,000	33,000	37,000
1,200-Ft. Site				
Settlement due to 210 psf design load (in)	11.0	4.5	2.0	---
Bearing capacity (psf)	230	345	490	---
Lateral resistance (lbs)	4,400	15,000	27,000	---

* Defined as that time after which no further improvement in soil properties occurs; the duration depends largely upon original soil properties and may require over one year under some conditions.

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13. ABSTRACT

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Soil mechanics						
Performance tests						

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Unclassified

Security Classification